Field Performance of Vertical Drain Installed in Ariake Clay Deposit

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Abstract. The test embankments on Saga airport site were back analyzed to evaluate the field performance data of prefabricated vertical drain (PVD) and sand drain (SD). It is emphasized that to back evaluate the field effect of vertical drain using correct parameters for natural subsoil is important. In this study, the soil parameters were verified by using the measured data of test embankment on natural subsoil. With fixed soil parameters, the field performance data of drain were back calculated by fitting the observed data. Based on back calculated results, it is suggested that for PVD installed in Ariake clay deposit, a discharge capacity of less than 100 m^3/year can be used for design. The smear zone diameter (d_s) of 3.0 times of equivalent mandrel diameter, and the smear zone permeability of 1/10 of field value of undisturbed zone are proposed. For sand drain, the sand permeability from laboratory test can be used in design providing to consider the possible reduction due to non-uniformity of sand used and partial saturation effect in field. The smear effect can be considered with the same way as suggested for PVD case. Also, for the case studied, a maximum hydraulic gradient of 0.3 within the drain was obtained from numerical results.

1. Introduction

The deposit in Saga plain mainly consists of soft Ariake clay, which has low strength, high sensitivity, and high compressibility. Normally, the ground improvement is required for geotechnical engineering activities in this region, such as road and airport constructions. Improving the engineering properties of the soft clay by vertical drain with pre-loading is an economic and effective method, and it has been applied to several projects, such as Saga airport construction.

The design of vertical drain improvement is usually based on unit cell theory (Baron, 1948; Hansbo, 1981). In unit cell theory, the factors affecting the drain behavior are: (1) spacing, (2) smear effect, and (3) well resistance (discharge capacity). At present, due to uncertainties related to parameter determination, the field performance of vertical drain can not be well predicted. In several cases, the field effect of vertical drain was overpredicted and the expected improvement could not be achieved (Chai et al., 1996). The most reliable way of evaluating the field behavior of vertical drain is by back analyzing the existing case histories. The back evaluated field performance data can form the base for improving design method or the accuracy of prediction. Moreover, by comparing the back analyzed values with laboratory data, the laboratory test technique can be advanced.

In Saga airport site, three test embankments were constructed to verify the effect of vertical drain improvement. One embankment was on natural subsoil to serve as reference, one was on prefabricated vertical drain (PVD) improved subsoil, and another was on sand drain (SD) improved subsoil. For easy to compare, all embankments had the same geometry. A unique set of measured data was obtained from these test embankments including settlements, lateral displacements, and excess pore pressures in the subsoils, which can be used to evaluate the field performance of vertical drain in Saga plain.

In this paper, firstly, the method used for modelling the vertical drain effect in plane strain analysis and the methodology of back evaluating the drain behavior are discussed. Then, the test embankments on Saga airport is described. Finally, the back analyzed field performance data of vertical drain (PVD and SD) are presented. The discussions are also made on determining the mechanical properties of both natural subsoil and vertical drain.

2. Method of Analysis

2. 1 Modelling vertical drain effect in plane strain condition

For test embankment on soft ground, there are two aspects which differ from unit cell assumptions, namely (1) the deformation of ground under embankment loading is not a 1-dimensional but 2 or 3-dimensional problem, and (2) the natural deposit is not uniform but layered. Therefore, to analyze the performance of test embankment, a numerical method is required. In this study, plane strain finite element method was used.

An individual drain is working in an axisymmetric condition. In plane strain analysis, to consider the effect of drain, it is necessary to model the axisymmetric drainage condition to plane strain one. To obtain the same average degree of consolidation in plane strain case as that in axisymmetric one, either permeability of soil, or spacing, or discharge capacity of drain should be modified. The modelling methods have been reviewed by Chai et al. (1995), and Chai and Miura (1997). In this study, the method proposed by Chai et al. (1995) was used. The method employs a 1-dimensional drainage element to model the effect of vertical drain and the axisymmetric condition is matched in plane strain case by modifying the well resistance of vertical drain.

2. 2 Methodology used for back analysis

For embankment on vertical drain improved subsoil, the observed performance data reflect (1) the properties of natural subsoil and (2) the effect of vertical drain. In back analysis, the mechanical parameters, especially the permeability of subsoil should be verified first. As shown in Fig. 1, the field measured data of improved ground is known, and the effect of drain is the difference between the measured data and estimated corresponding values in the case of without drain. If the permeability of natural ground is under estimated, the drain effect will be over-evaluated. This is most possibly to occur because, normally, laboratory test under-estimates the field value of permeability (Tavenas et al., 1986)). In the analysis, first, the parameters for natural subsoil were verified by comparing the measured and simulated data of the embankment on natural subsoil. Then, the parameters for natural subsoil were fixed, the parameters for vertical drain were verified to fit the observed field data to evaluate the field performance of vertical drain.

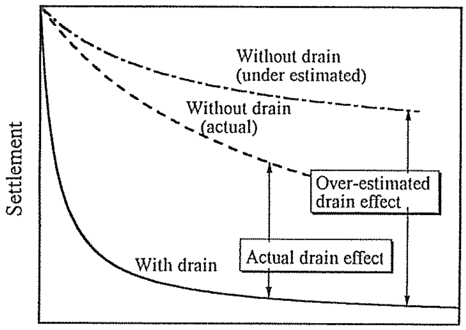


Fig. 1 Sketch illustrating the effect of vertical drain on settlement

Uncertainty parameters related to drain behavior are: (1) well resistance (discharge capacity), and (2) smear effect represented by the diameter of smear zone (d_s) and the permeability ratio of the horizontal permeability of natural soil to that of smear zone (k_h/k_s). These factors are independent, and it is impossible to determin-

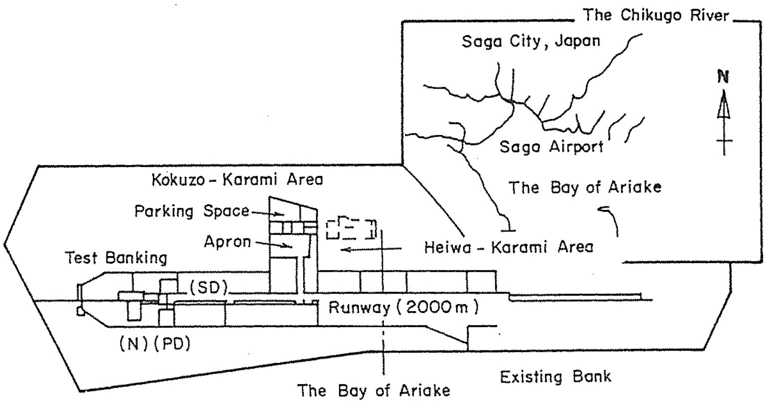
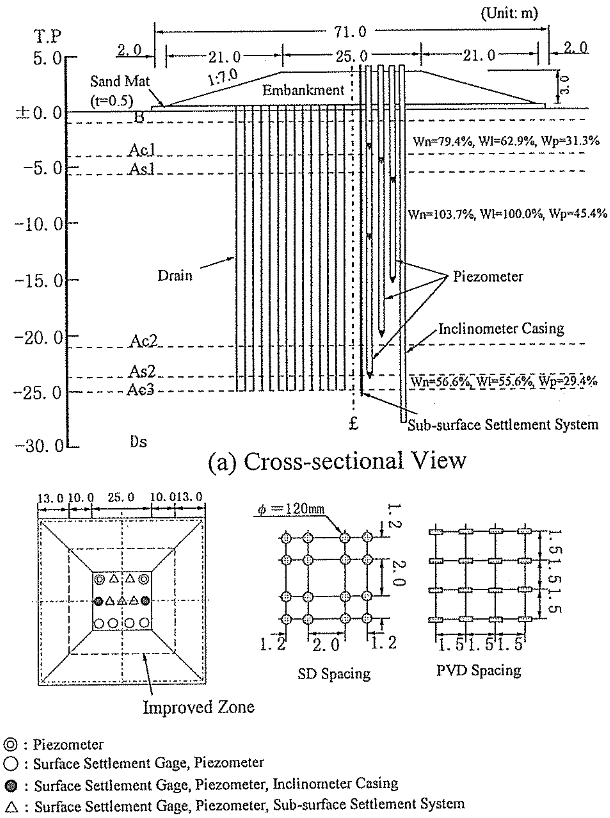


Fig. 2 Location of Saga Airport

ing all of them from a single analysis. The method adopted was that: (1) to estimate a set of base parameters from laboratory test data, (2) to vary one of the parameters at one time to fit the observed data and establish a possible variation range of each parameter, and (3) to apply some engineering judgements to get the best estimated field values.

3. Full Scale Test Embankments at Saga Airport



(b) Plane View

Fig. 3 Geometry of embankment and the location instrumentation points

The test embankments on Saga Airport were reported by Bergado et al. (1996), and a brief description is given here. Saga Airport is located at 13 km south of Saga city on a reclaimed land close to Ariake sea. The soft deposit is about 25 m thick consists of 3 clay layers and 2 thin sand layers. At Kakuzo-Karami side, 3 test embankments were constructed on natural, PVD improved and SD improved subsoil (Fig. 2). The embankments had the same geometry with a fill thickness of 3.5 m, base width of 71 m by 71 m, and top width of 25 m by 25 m. The filling speed was about 0.03 m/day. The PVD and SD were installed to around 25 m depth and the improved zone was under the center of the embankment with a width of 45 m by 45 m. PVD was installed in a square pattern with a spacing of 1.5 m. SD was installed also in a square pattern with an average spacing of 1.6 m. At one time, 4 drains in a group were installed with a spacing between the drains of 1.2 m. The spacing between groups was 2.0 m. Fig. 3 shows the geometry of the embankments, main instrumentation points, the pattern of drain installation, and the soil profile of PVD section.

The thickness of the soil layers is slightly varied at three embankment locations. Top weathered crust (B) is about 1.0 m thick followed the first soft clay layer, Ac1, with a thickness of 2.5 to 3.0 m. A sand layer, As1, of 1.1 to 1.5 m thick underlies Ac1. The main clay layer, Ac2, below As1, is very soft with a thickness of 13.5 to 15.5 m. Under Ac2 is a sand layer, As2, which has a thickness of 0.5 to 2.7 m. The third clay

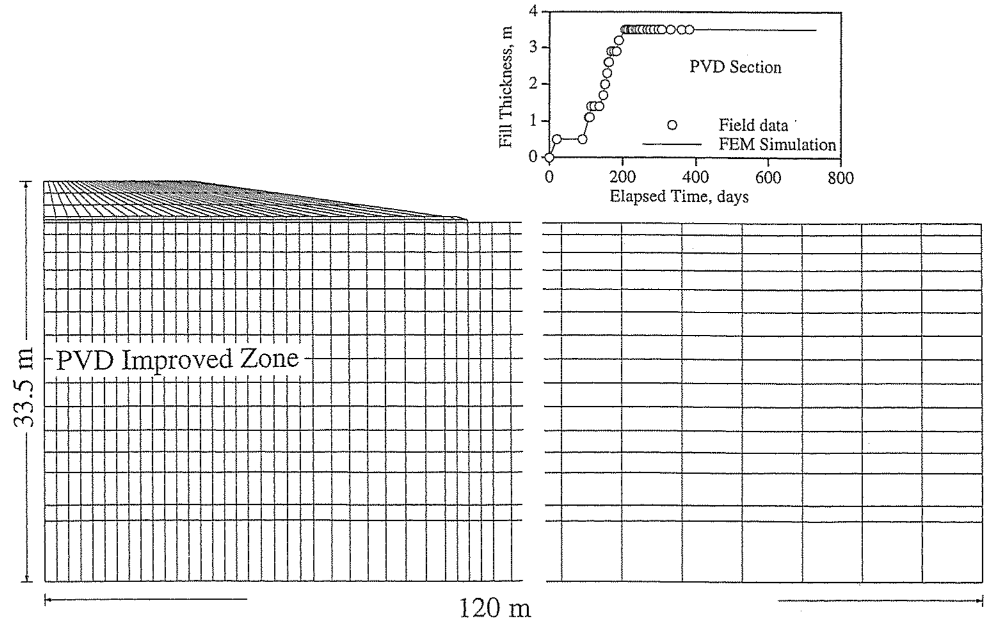


Fig. 4 Finite element mesh and construction history for PVD section

layer, Ac3, is soft to middle stiff and has a thickness of 1.3 to 3.6 m underlying a thick and dense sand layer (DS).

4. Finite Element Analysis

4. 1 Boundary conditions and model parameters

In finite element analysis, plane strain condition was assumed. The modelled range was 30 m depth from ground surface, and horizontally 120 m away from embankment centerline. The displacement boundary conditions were: at bottom, both vertical and horizontal displacements were fixed; for left and right vertical bound-

Table 1 Model Parameters for Subsoil

Layer	Young's Modulus E (kPa)	Poisson's Ratio v	κ	λ	M	Initial Void Ration e_0	Unit Weight γ (kN/m³)	K_x (10^{-8} m/s)	K_v (10^{-8} m/s)
B		0.25	0.025	0.25	1.3	2.00	15.0	11.45	7.60
Acl		0.30	0.044	0.44	1.2	2.00	14.5	5.7	3.8
Asl	10,000	0.20					15.5	290	290
Ac2		0.30	0.087	0.87	1.2	2.50	14.5	2.64	1.76
As2	15,000	0.20					16.0	290	290
Ac3		0.30	0.030	0.30	1.3	1.75	16.0	2.64	1.76
Ds	30,000	0.20					19.0	290	290

λ : virgin loading slope in e-ln (p') plot (p' is effectitve mean stress).
κ : reloading/unloading slope in e-ln(p') plot.
M : slope of failure Iine in p' versus q plot (q is deviator stress).

aries, the horizontal displacement was fixed. The adopted drainage boundary conditions were: ground surface and bottom line (sand layer) were drained; the left and right boundaries were undrained, but for right boundary (away from embankment centerline), at the location of sand layer was drained. Fig. 4 shows the finite element mesh for PVD section, and the construction history is also indicated in the figure. For the zone with vertical drains, one-dimensional drainage elements coincide with every other vertical line.

The mechanical behavior of clay layers was represented by modified Cam clay model (Roscoe and Burland, 1968) and the sand layers as well as decomposed granite fill material were assumed as elastic. The determined model parameters for subsoil are listed in Table 1. For clay layers, the compression parameters, κ and λ, and initial void ratio, e_0, were determined from laboratory consolidation tests (Bergado et al., 1996), and the slope of failure line in p'-q plot (p' is effective mean stress, and q is deviator stress), M, was obtained from triaxial test results. Poisson's ratio, ν, was assumed empirically. For the value of permeability, two things need to be defined, namely the initial value as well as the way of its variation during consolidation process. The coefficient of consolidation is a function of both permeability of soil and the stiffness of soil. In coupled finite element analysis, instead of using the coefficient of consolidation, the permeability and stiffness are used independently. With the increase of degree of consolidation, the stiffness of soil will increase. If the value of permeability is fixed, the coefficient of consolidation will increase during the process of consolidation. However, the experiment evidences indicate that the coefficient of consolidation is constant or reducing during consolidation. Therefore, considering the permeability variation is important for making a correct prediction. It has been understood that the value of permeability varies with void ratio, and the equation proposed by Taylor (1948) is widely used.

$$k = k_0 10^{[-(e_0-e)/C_k]}$$

where e_0 is initial void ratio, e is current void ratio, k_0 is the initial value of permeability, k is the current value of permeability, and C_k is constant, which can be estimated as $0.4e_0$ to $0.5e_0$ (e.g. Tavenas et al., 1983). For the value of initial permeability, first the ratio of horizontal value to vertical value (k_h/k_v) was determined from laboratory test results (Park, 1994) as 1.5. Then, the initial values of vertical permeability were adjusted to fit the measured data of the embankment on natural subsoil, and which were about 4 times of the value from laboratory consolidation

Table 2 Initial stress for Ariake clay deposit and size of yield locus (PVD section)

Depth m	σ_{x0} kPa	σ_{y0} kPa	u_0 kPa	P_y kPa
0.0	5.0	0.0	0.0	29.1
1.0	9.6	15.5	0.0	29.1
4.0	18.0	29.0	30.0	30.3
5.6	20.8	37.8	46.0	40.3
21.0	53.6	107.1	200.0	95.2
23.7	61.7	123.3	227.0	109.6
25.0	65.6	131.1	240.0	143.1
30.0	88.1	176.1	290.0	184.4

Note: σ_{x0}-initial horizontal stress, σ_{y0}-initial vertical stress, u_0-pore water pressure; P_y- size of yield locus

Table 3 Parameters Related to Drain Behavior

Item	Symbol	Unit	Values	
			PVD	SD
Drain diameter	d_w	mm	48.3	120
Unit cell diameter	D_e	m	1.7	1.8
D_e/d_w	n		35.2	15
Smear zone diameter	d_s	mm	300	412
Permeability ratio	K_h/K_s		10	9
d_s/d_w	s		6.2	3.4
Discharge capacity	q_w	m^3/year	75-500	35-265

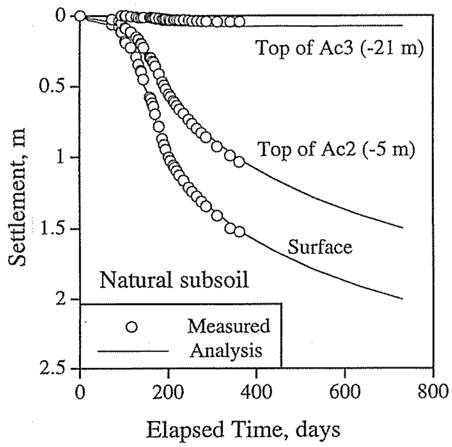


Fig. 5 Comparison of settlement for embankment on natural subsoil

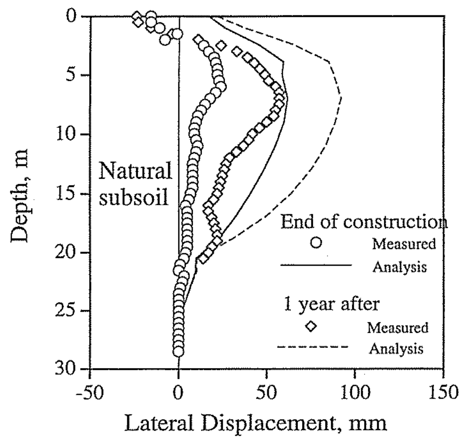


Fig. 6 Comparison of lateral displacement for embankment on natural subsoil

test result. Young's modulus for sand layer was estimated by referring to standard penetration test results. The values of permeability of sand layers were assumed. Finally, the mechanical properties of fill material were assumed as: Young's modulus of 15,000 kPa and Poisson's ratio of 0.2. The unit weight of fill material was 20 kN/m^3.

The subsoils were in a lightly over-consolidated to normally consolidated states with a maximum over consolidation ratio (OCR) of about 4 for top crust. The lateral earth pressures were calculated using the equation proposed by Mayne and Kulhawy (1982). The ground water level was about 1.0 m below ground surface. Table 2 lists the estimated initial stress and the size of yield locus of subsoil under PVD improved section.

The parameters for PVD and SD are given in Table 3. The diameter of smear zone was estimated as 3.0 times of equivalent (by area) mandrel diameter (Miura et al., 1993). There are many uncertainties regarding the value of k_h/k_s, which is a function of the structure and sensitivity of subsoil. Since for most natural deposit, the permeability in horizontal direction is higher than in vertical direction. Hansbo

(1987) proposed that k_s can be the same as vertical permeability of natural soil, k_v. The value of k_h/k_v can vary from 1 to 15 (Jamiokowski et al., 1983). The higher permeability in horizontal direction mainly due to the existing of sand seams or very thin sand layers. However, this effect is difficulty to be measured in laboratory due to sample disturbance and sample size effect. It is argued that laboratory test may be a correct way for determining k_s value, but it generally under-estimates the permeability of field deposit. It is suggested that the field k_h/k_s value can be expressed as the laboratory value multiplied by a ratio, C_f, field value of permeability over the laboratory value. As mentioned previously, at Saga airport site, C_f was about 4. The smallest value of permeability in inner smear zone for Ariake clay was reported as 1/5 of that undisturbed zone from laboratory test (Madhav et al., 1993). If assuming a linear variation of permeability in smear zone, a single value of representative permeability in smear zone (equal smear effect) (Chai et al., 1997) from laboratory test is 1/2.5 and 1/2.3 of that undisturbed zone for PVD and SD case, respectively. The values in table 3 are the laboratory values (2.5 and 2.3) multiplied by 4 (C_f). The base value of discharge capacity for PVD was from laboratory test results of confining the drain in clay and tested for 5 month (Miura et al., 1998), and the value for SD was computed by using the laboratory value of permeability of the sand used (1.0 to $7.4 \times 10_{-4}$ m/s).

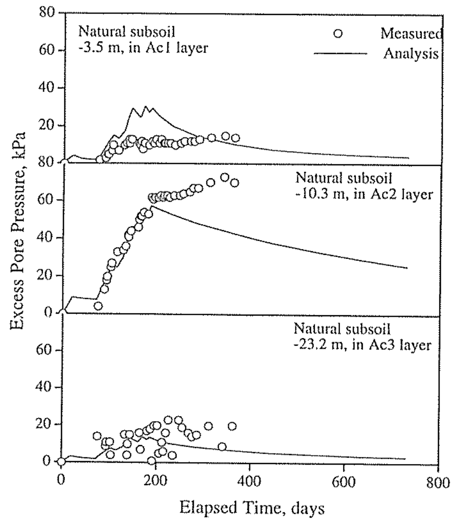


Fig. 7 Comparing the excess pore pressure for embankment on natural subsoil

4. 2 Analysis results

(1) Embankment on natural subsoil. The

Table 4 Back Calculated Parameters Regarding to Drain Behavior

(1) Fix K_h/K_s and d_s, back calculate q_w			
Case	k_h/k_s	d_s	q_w (m^3/year)
PVD	10	$3d_m$	85
SD	9	$3d_m$	100
(2) Fix d_s and q_w, back calculate k_h/k_s			
Case	k_h/k_s	d_s	q_w (m^3/year)
PVD	11.5	$3d_m$	500
SD	10	$3d_m$	265

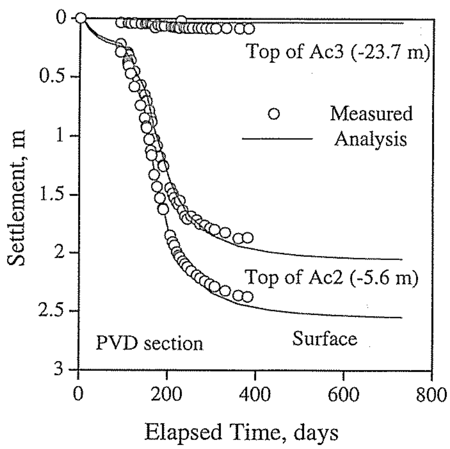


Fig. 8 Comparison of settlement for PVD section

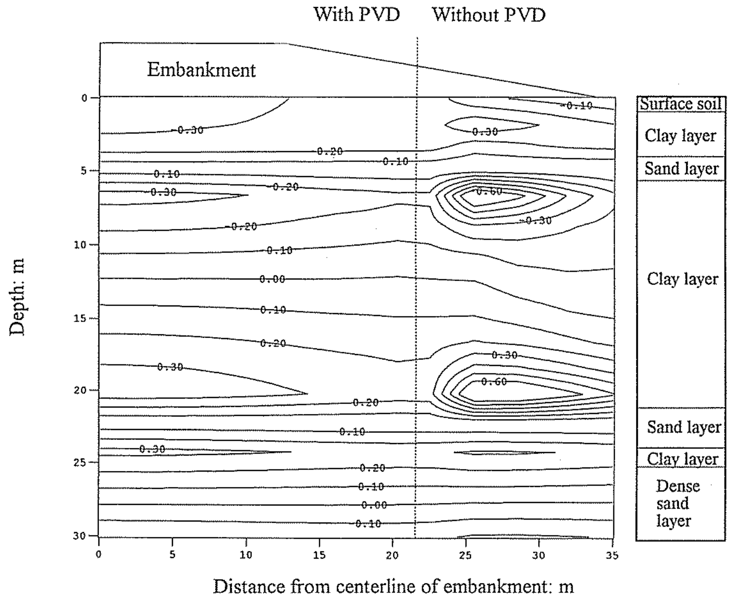


Fig. 9 Hydraulic gradient contour at the end of construction (PVD section)

purpose of analyzing the embankment on natural subsoil is to verify the model parameters as well as numerical procedure. The numerical results are compared with measured data in Figs. 5 to 7 for settlements, lateral displacements, and excess pore pressures, respectively. As can be seen from Fig. 5, the analysis simulated the settlement curve well. The comparisons for lateral displacement and excess pore pressure are fair (Figs. 6 and 7). For the piezometer point in Ac2 layer, after embankment construction, the measured excess pore pressure continuously increased, and the reason is not clear. The comparisons indicate that the adopted parameters for natural soils and modelling method are acceptable.

(2) Field performance data of PVD. To fit the measured data, either the discharge capacity of the drain or the permeability in smear zone was varied, and the back calculated values are listed in Table 4. It can be seen that for fixing smear effect case, a discharge capacity of about 85 m^3/year was obtained, which almost matches the laboratory long term value of confining the drain in clay. If using the lower laboratory value of discharge capacity, 75 m^3/year in Table 3 (5 month data), the best estimated k_h/k_s value was almost the same as estimated base value. If using a higher

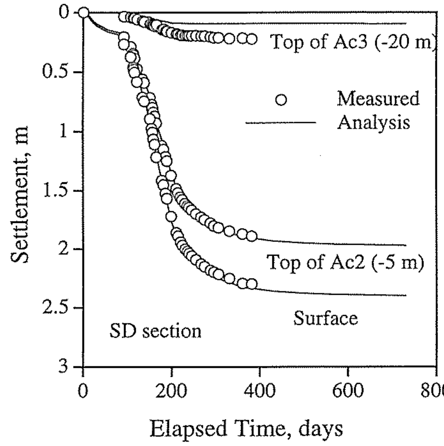


Fig. 10 Comparison of settlement for SD section

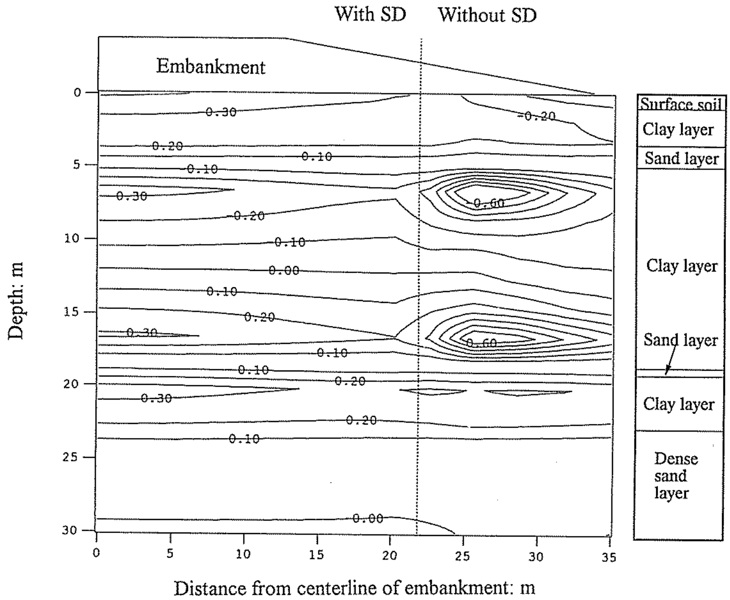


Fig. 11 Hydraulic gradient contour at the end of construction (SD section)

value of discharge capacity, 500 m^3/year in Table 3 (1 week data) (Miura et al., 1998), the ratio, $k_h/k_s = 11.5$ was resulted.

In back analysis, the target was settlement, the comparison of settlements on embankment centerline is presented in Fig. 8. It shows that the numerical results simulated the field data well. However, if checking carefully, it can be noticed that the analysis slightly under-predicted the settlement during construction period and slightly over-estimated the settlement for after construction period. One of possible reasons is the continuous variation (reduction) on discharge capacity of PVD with elapsed time, i.e. the initial discharge capacity might be larger than 85 m^3/year, and the value might be less than 85 m^3/year during consolidation period. The back estimated field discharge capacity was under the assumption that the discharge capacity is a constant during consolidation. If the relation between discharge capacity and elapsed time can be established, it can be incorporated into numerical procedure to improve the accuracy of prediction. The comparisons of lateral displacement and excess pore pressure are similar to that of embankment on natural subsoil.

Hydraulic gradient (i) within PVD was also calculated from numerical results and shown in Fig. 9 in contour form for the end of construction case. It indicates that the highest i value is about 0.3. With the dissipation of excess pore pressure, i will gradually reduce. It is considered that the field hydraulic gradient within drain is useful for determining a proper i value for discharge capacity test. Of course, i value will vary with loading speed, ground condition, etc. and Fig. 9 just provides a reference on this aspect.

(3) Field performance data of SD. For SD case, the back calculated discharge

capacity is within the range of laboratory value (Table 4). This result suggests that for SD method, the laboratory value of sand permeability can be directly used for design by considering the possible reduction of permeability caused by: (1) non-uniformity of sand used, and (2) partial saturation effect in field. If fixing q_w as 265 m^3/year (upper value of sand permeability), a value of $k_h/k_s = 10$ was obtained, which is not different much from base value.

The settlement on embankment centerline is compared in Fig. 10. For this case, the analysis simulated settlement curve very well. Comparing with PVD case, there is no over-prediction after construction for SD case, which indicates that the drainage capacity of SD may not change much during the consolidation process of subsoil.

The same as for PVD section, from numerical results, a maximum hydraulic gradient of about 0.3 within sand drain was obtained corresponding to the end of construction condition (Fig. 11). It can be used as a reference for selecting a proper hydraulic gradient for laboratory permeability test of sand.

5. Conclusions

The test embankments on Saga airport site were back analyzed to evaluate the field performance data of PVD and SD. It has been reasoned that verifying the mechanical properties of subsoil is very important for correctly estimating the field effect of drain. In this study, it has been done by using the measured data of the test embankment on natural subsoil. Based on the back calculated field performance data, following suggestions can be made on design vertical drain improvement in Saga plain.

(1) For PVD improvement, the discharge capacity of less than 100 m^3/year can be used. For smear effect, the smear zone diameter $d_s = 3.0 d_m$ (equivalent mandrel diameter), and $k_h/k_s = 10$ are suggested.

(2) For SD case, considering the non-uniformity of sand used and the partial saturation effect in field, the lower value of laboratory permeability test results on sand need to be used for design. The smear effect needs to be considered with the same way as suggested for PVD case.

For the case investigated, the hydraulic gradient within the drain was calculated from numerical results, and a maximum value of 0.3 was obtained corresponding to the end of construction condition.

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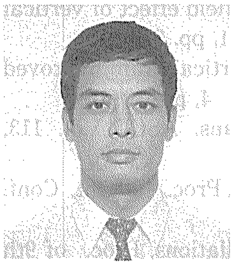

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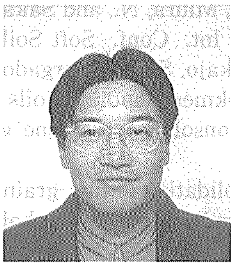

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